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ILLICON Analysis of Ellingsrud Test Fill

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Synopsis: The 50-m diameter test fill at Ellingsrud, Norway was constructed by NGI to determine preconsolidation pressure of a soft clay, mobilized under field conditions. The height of the fill was increased in four increments during 1972 to 1978. Settlement observations as well as pore water pressure measurements over a period of eleven years are used to evaluate predictions made by the ILLICON computer program. ILLICON analysis requires vertical profiles of compressibility and permeability parameters.

The computed surface settlements are within 2 cm of the measured values during the first three loading stages. After the fourth loading, there is a larger range in the measured settlements, however, the computed values lie within the measured range. The computed pore water pressures are also in reasonable agreement with the observations. However, compressibility and permeability data for major depth intervals of the 24 m soil profile are not available. Therefore, in spite of good agreement between prediction and field observation one can not be completely certain whether the input parameters in fact represent the true condition of the soil.

Introduction

In the fall of 1972, a circular test fill was constructed by the Norwegian Geotechnical Institute, at Ellingsrud outside Oslo, to determine the in situ preconsolidation pressure of a soft clay deposit. Bjerrum was interested to compare and establish further correlations between σ_p/σ_{vo} and plasticity index, and to check whether a clay which has become leached has lost some of the preconsolidation pressure which had previously been developed as a result of secondary compression. Construction of the 50-m diameter and 0.42 m high sand fill started on November 14, 1972 and was completed in 5 days. The height of the fill was increased every other year from 1972 to 1978. The unit weight of the sand fill was 17.6 kN/m^3 .

The test fill site is fairly flat and the clay deposit underlying the test fill is 24 m thick. The upper part of the clay layer is weathered and a stiff crust extends from the ground surface to 3 to 4 m depth. Near the ground surface, vertical cracks were found with some extending below the crust. The silty clay deposit below the crust is in turn underlain by the bedrock. This silty clay layer is very homogeneous and is characterized by high sensitivity. Its water content is about 30% right below the crust and at depths greater than 14 m. Between these depths the water content is about 40%. The plasticity index of the silty clay is only 3 to 6%. A typical soil profile is shown in Fig. 1.

The water table is near the clay surface. Piezometer observations in the natural ground indicate hydrostatic condition within the fissured crust. However, a steady seepage condition, corresponding to an overpressure of about 2 m of water in the bedrock, exists within the silty clay layer.

Field instrumentation included settlement markers and piezometers. Under the fill, settlement markers of various types were installed on the ground surface and at 2 m depth. Undisturbed samples were taken by means of 95 mm fixed piston samplers in 1972 and 4 block samples were taken in 1983 using Sherbrooke block sampler (Lefebvre and Poulin, 1979).

Analyses of settlement and pore water pressure behavior were made using ILLICON, starting from the first loading. ILLICON is a methodology for one-dimensional settlement analysis, including the procedures for obtaining the compressibility and permeability parameters (Mesri and Choi, 1985b), and it is only explained here in relation to selecting input parameters for the analysis of the Ellingsrud test fill.

In situ Void Ratio Profile

One-dimensional settlement analysis is based on the assumption that settlement results from a decrease in void ratio. In situ void ratio, e_o , defines the initial condition.

All of the water content data collected by NGI, including those corresponding to the 95 mm piston samples and block samples on which consolidation tests were performed, were utilized in establishing the e_o profile shown in Fig. 2. Void ratios were computed from the water content data using a specific gravity of solids of 2.80, and assuming a fully saturated clay. There were no water content data below the depth of 18 m. But a large number of vane borings showed that the clay deposit is very homogeneous down towards the rock. Based on the vane shear strength data and the information that the soil just above the bedrock contains sand and gravel, the water content of this layer is believed to be lower than the rest of the deposit. Based on the profiles of e_o , σ_p and k_{vo} , the clay layer was divided into 10 sublayers. The values of e_o selected for the sublayers, including the assumed void ratio profile for depths below 18 m, are shown in Fig. 2.

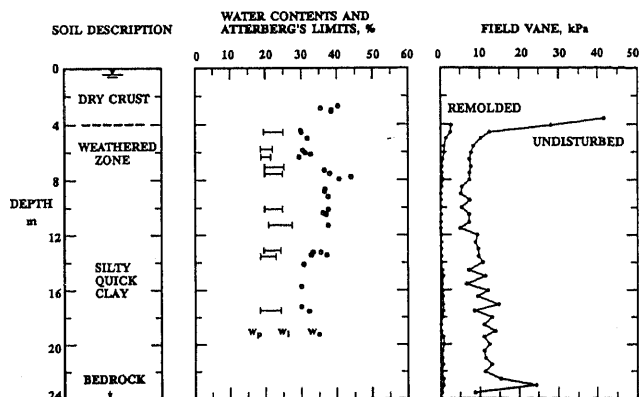


Fig.1 Soil Profile Under Ellingsrud Test Fill

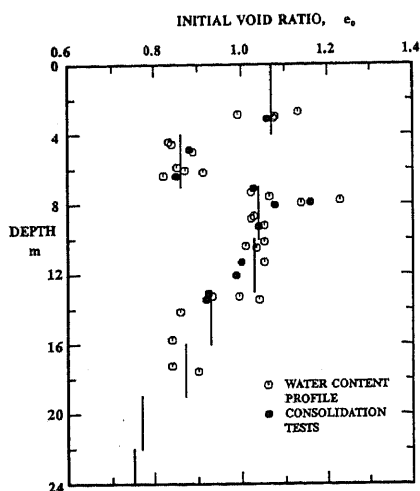


Fig. 2 Initial Void Ratio Profile

Initial Effective Vertical Stress

The only stress involved in one-dimensional settlement analysis is the effective vertical stress. Pre-construction effective overburden pressure, σ_{vo} , defines the initial condition. The σ_{vo} profile shown in Fig. 3 was computed using the values of unit weight reported by Karlsrud and Haugen (1979) and assuming a water table 0.2 m below ground surface. For the soil below 14 m depth where unit weight data were not available, a value of 19.2 kN/m^3 , which is the unit weight of the soil immediately above it, was assumed.

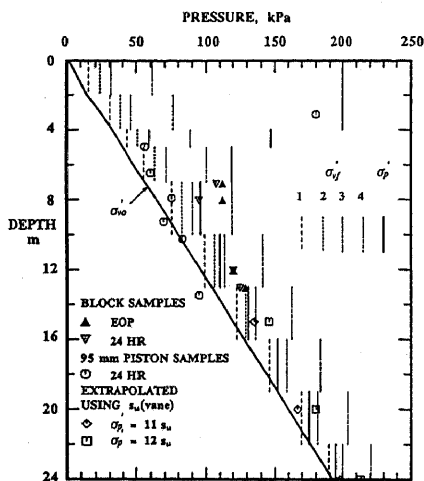


Fig. 3 σ_{vo} , σ_p and σ_{vf} Profiles

Preconsolidation Pressure

One of the most important parameters in one-dimensional settlement analysis is the preconsolidation pressure, σ_p , which defines the boundary between the recompression range where compressibility is small and the compression range in which compressibility is significantly larger. The ILLICON approach, assumes the uniqueness of the end-of-primary (EOP) $e - \log \sigma_v$ relationship (Mesri and Choi, 1985a). In other words, it is

assumed that EOP $e - \log \sigma_v$ curves from laboratory tests can be used directly for settlement analysis of the sublayers in situ. Therefore, the EOP σ_p profile is required as an input parameter to consolidation analysis.

Preconsolidation pressures determined from the oedometers tests on 95 mm piston samples and block samples are shown in Fig. 3. The values of σ_p of the 95 mm samples are smaller than those of the block samples, in some cases they are even smaller than the corresponding effective overburden pressure. It is believed that the 95 mm samples were disturbed (Lacasse et al. 1985). Hence, only the σ_p derived from block samples were used. From the compression-time data of incremental loading oedometer tests on block samples, the EOP $e - \log \sigma_v$ curves were constructed and values of σ_p were obtained. The EOP σ_p of Ellingsrud clay are generally very close to the 24 hr σ_p . For the soils below the depth of 14 m where there were no compressibility data, the relationship between the field vane undrained shear strength and the preconsolidation pressure was used to extrapolate σ_p data. An average line was drawn through the field vane strength profile between the depths of 12 m and 24 m. Values of vane strength at the depths of 12 m and 13 m were read off from this line and were divided by the corresponding EOP σ_p . The values of s_u/σ_p range between 1/12 and 1/11. Using these ratios and the vane strength profile, data on σ_p were generated for sublayers below 14 m depth. The values of EOP σ_p used in the consolidation analysis are shown in Fig. 3.

Final Effective Vertical Stress

The magnitude of final primary settlement depends on the magnitude of the final effective vertical stress, σ_{vf} . The value of σ_{vf} at middepth of each sublayer is computed from $\sigma_{vf} = \sigma_{vo} + \Delta\sigma_v$. For the consolidation analysis of the Ellingsrud test fill, $\Delta\sigma_v$ profiles under the center of a 50-m diameter circular fill for each loading stage were computed using the Boussinesq elastic stress distribution. The values of σ_{vf} in Fig. 3 show that only after the third loading did σ_{vf} exceed σ_p in all five bottom layers.

EOP $e - \log \sigma_v$ Curves

In the ILLICON analysis, an EOP $e - \log \sigma_v$ curve for each sublayer is input into the computer. The EOP $e - \log \sigma_v$ curves from laboratory consolidation tests can be directly used for this purpose. However, the ILLICON approach recommends a procedure for constructing the most representative EOP $e - \log \sigma_v$ for each sublayer. This approach is especially useful when: (a) Either a large number of consolidation tests have been performed and more than one $e - \log \sigma_v$ curve corresponds to each sublayer, or very few consolidation tests and $e - \log \sigma_v$ curves from a limited number of depths are available. In the first case, the approach is used to average large amount of data, while in the second it is used to extrapolate limited information; (b) Additional data on σ_p profile are available for example from empirical correlations, or from related subsurface investigation; (c) The e_p profile is better defined using all the available water content data rather than only those from limited number of consolidation specimens; and (d) The values of recompression index, C_r , are established from either empirical correlation or previous experience.

The compression EOP $e - \log \sigma_v$ curves, which are generally non-linear, are expressed in terms of C_c using the procedure described by Mesri and Choi (1985b). The EOP $e - \log \sigma_v$ curves from laboratory tests are used to define a set of $C_c - \log \sigma_v/\sigma_p$ relationships corresponding to either different in situ void ratios or depths. The most representative EOP $e - \log \sigma_v$ curve for each sublayer starts from point e_o , σ_{vo} (data from Figs. 2 and 3), recompresses with a constant C_r until σ_p (data from Fig. 3) is reached. Beyond σ_p , the non-linear compression $e - \log \sigma_v$ is constructed using the C_c data.

For Ellingsrud test fill, seven oedometer tests were performed on 95 mm piston samples and four tests on block samples. Lacasse et al. (1985) report that the 95 mm piston samples "show more signs of disturbance, with lower σ_p -values and higher recompression index during first loading". The $e - \log \sigma_v$ curves for piston samples were substantially below the ones from block samples at similar depths. Hence, only the $e - \log \sigma_v$ curves from block samples were used in the analysis. Therefore, conditions (a) through (d) are applicable to the $e - \log \sigma_v$ data for Ellingsrud test site as the $e - \log \sigma_v$ curves for a major part of the silty clay profile are not available. It is necessary to extrapolate C_c from different depths. Based on the oedometer test results on block samples, the value of C_r for Ellingsrud clay was estimated to be 0.017. The resulting EOP $e - \log \sigma_v$ curves of the sublayers used in the consolidation analysis are shown in Fig. 4. Each EOP $e - \log \sigma_v$ was input into the computer using up to 15 (e, σ_v) data points.

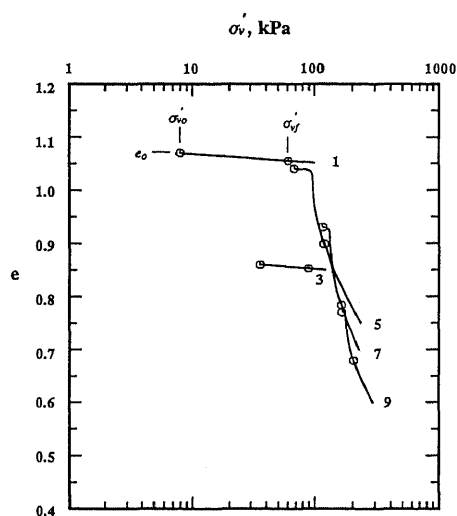


Fig. 4a EOP $e - \log \sigma_v$ of Sublayers

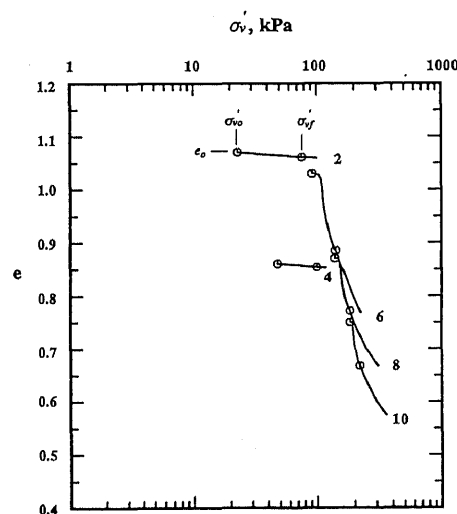


Fig. 4b EOP $e - \log \sigma_v$ of Sublayers

Compressibility With Respect to Time

The ILLICON approach recognizes that after the completion of primary consolidation at σ_{vf} , compression continues with time. Secondary settlement is evaluated using the secondary compression index, C_a . Compressibility with respect to time also operates during the primary consolidation stage. The measured EOP $e - \log \sigma_v$ already includes this latter component, and is taken into account in formulating the constitutive equation (Mesri and Choi, 1985b). The only input parameter for including the effect of compressibility with time during the primary consolidation stage, and for computing secondary compression is, C_a/C_c . The corresponding values of C_a and C_c for Ellingsrud clay were computed using the consolidation data on block samples reported by Karlsrud and Haugen (1979). Based on these data which are shown in Fig. 5, a value of $C_a/C_c = 0.044$ was used in the consolidation analysis.

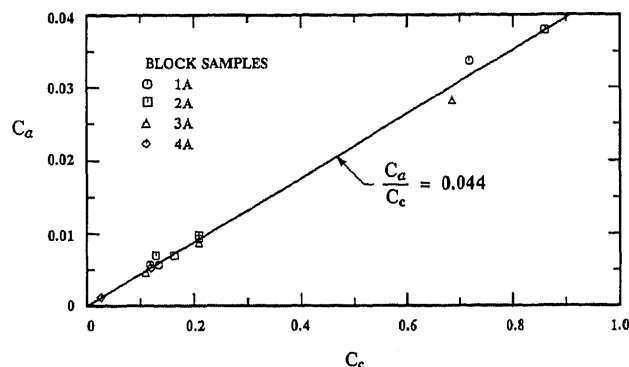


Fig. 5 Data on C_a/C_c

Coefficient of Permeability

ILLICON uses a finite strain hydrodynamic equation with a coefficient of permeability which decreases during consolidation. The vertical permeability of each sublayer starts at k_{v0} and decreases during compression according to the magnitude of C_k .

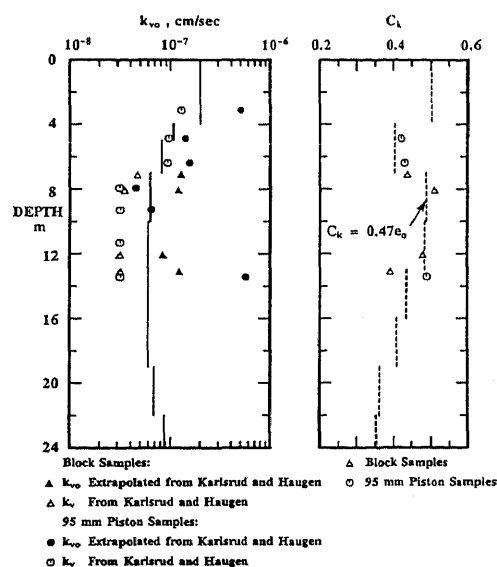


Fig. 6 k_{v0} and C_k Data for Ellingsrud Clay

For Ellingsrud clay, direct permeability measurements were made during oedometer tests on 95 mm piston samples and block samples. For each clay sample intended for permeability tests, two permeability measurements were carried out. From the vertical strain-log k_v plot, it was possible to extrapolate to zero compression and obtain k_{v0} . The results are shown in Fig. 6. The slope of vertical strain-log k_v plot was used to compute C_k .

For soils below 14 m depth where there were no permeability data, it is assumed that the permeability gradually increases as sand and gravel were found close to the bedrock. The permeability used for each sublayer is shown in Fig. 6. The (e_0, C_k) data points in Fig. 7 suggest $C_k = 0.47e_0$, which was used along with the e_0 profile in Fig. 2, to obtain the C_k profile in Fig. 6. It is interesting to note that this empirical correlation agrees very well with $C_k = 0.5e_0$, proposed by Tavenas et al. (1983). For the time rate of settlement analysis, the bedrock was assumed to be freely draining.

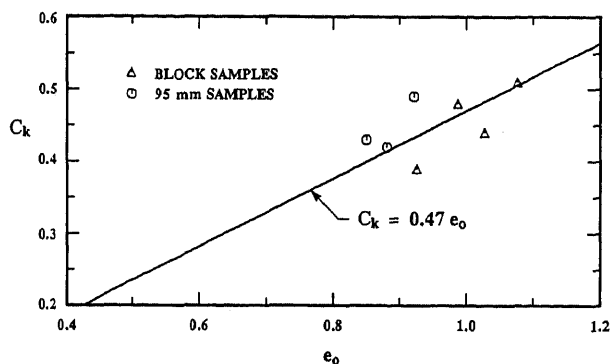


Fig. 7 Correlations between C_k and e_0

Observed and Computed Settlements

Two analyses were carried out, with one analysis accounting for the changes in applied pressure due to saturation and submergence of the fill material as consolidation of the subsoil progressed. The computed settlements are compared with the observations in Fig. 8. Settlements and pore water pressures were computed under the center of the fill. The observed settlement correspond to settlement markers at different radial

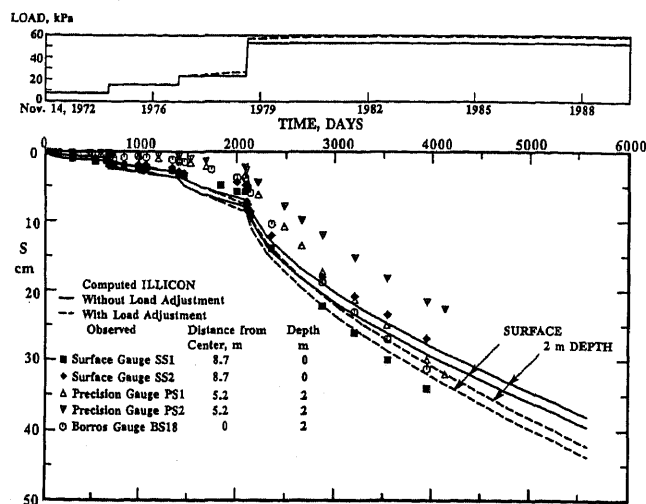


Fig. 8 Observed and Computed Settlement Behavior

distances from the center of the fill and at two different depths. The surface settlement gauges were located at the clay fill contact and at a radial distance of 8.7 m from the center of the test fill. All Borros and Precision settlement probes were installed at the depth of 2 m. One of the Borros settlement probes was located at the center of the fill. The Precision settlement gauges were at 5.2 m from the center of the fill.

The computed surface settlements exceed the observed settlement by no more than 2 cm during the first 3 loading stages, and are in general within 1 cm. This difference is partly due to the fact that the surface gauges were not located at the center of the fill. After the fourth loading stage, there is a large range in measured settlements by the surface gauges. The computed settlements lie within the range of the measured values. According to the predictions by ILLICON, final primary settlement of Ellingsrud test fill is 1.09 m, and 95 % primary consolidation will be reached in the year 2050 (Figs. 15 and 16)

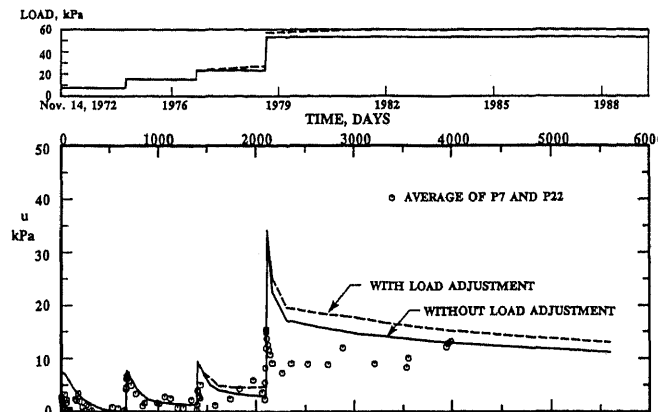


Fig. 9 Observed and Computed Excess Pore Water Pressure at 6 m Depth

Pore Water Pressure Behavior

ILLICON can accommodate any assumption on excess pore water pressure response upon loading. For the analysis of the Ellingsrud test fill, pore water pressure response was computed from $\Delta u = \Delta \sigma_v$, where $\Delta \sigma_v$ is obtained using elastic stress distribution. The time-dependent construction of the fill was also taken into account. The computed pore water pressure response shown in Figs 9 - 14, are in reasonable agreement with the observations, despite the fact that compressibility and

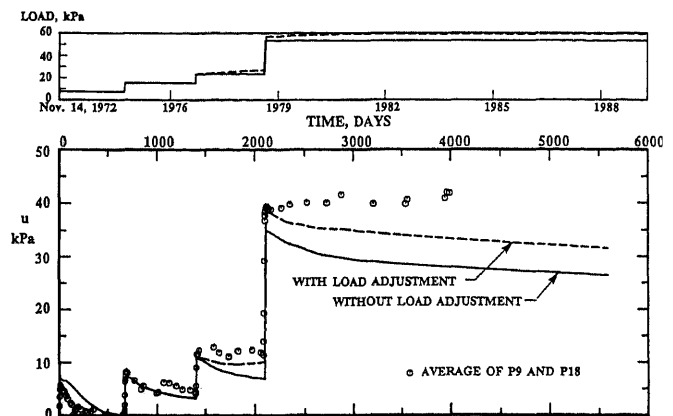


Fig. 10 Observed and Computed Excess Pore Water Pressure at 10 m Depth

permeability input data for a major part of the soil profile were not available. It can be seen that 3000 days after the final load is applied, there are still significant magnitudes of excess pore water pressure near the mid-depth of the clay deposit under the center of the fill. The slow rates of pore water pressure dissipation are believed to be caused by the high compressibility of the silty clays at stress levels slightly beyond the preconsolidation pressure.

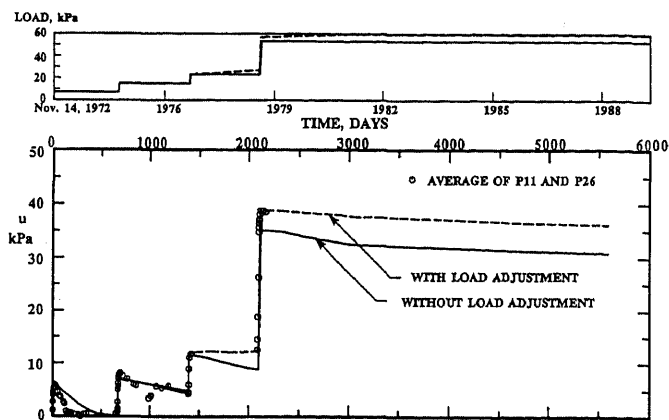


Fig. 11 Observed and Computed Excess Pore Water Pressure at 14 m Depth

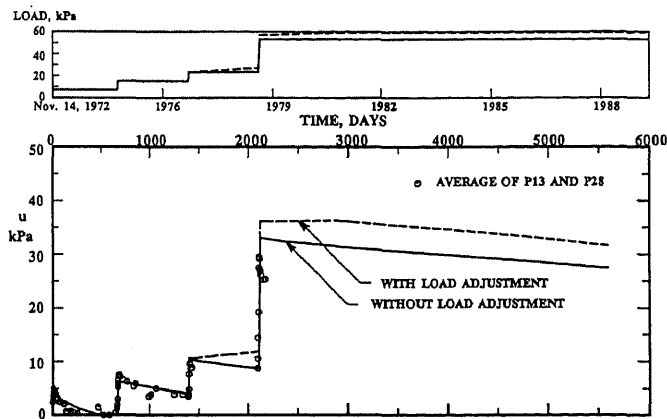


Fig. 12 Observed and Computed Excess Pore Water Pressure at 18 m Depth

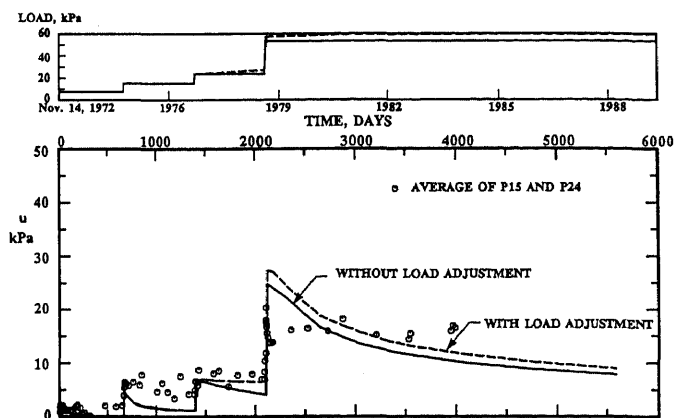


Fig. 13 Observed and Computed Excess Pore Water Pressure at 22.5 m Depth

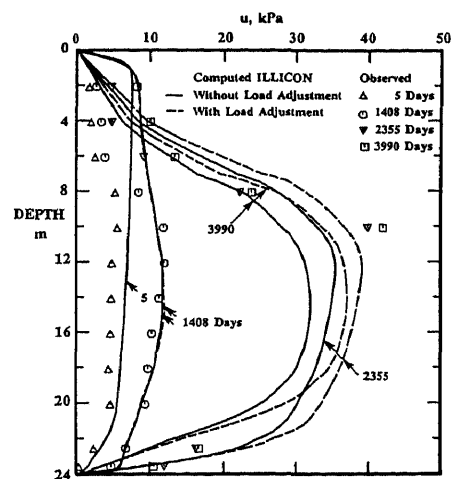


Fig. 14 Excess Pore water Pressure Isochrones

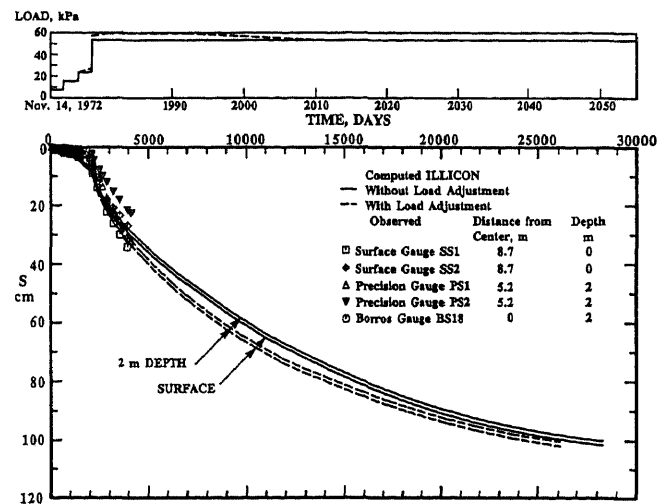


Fig. 15 Observed and Computed Settlement Behavior

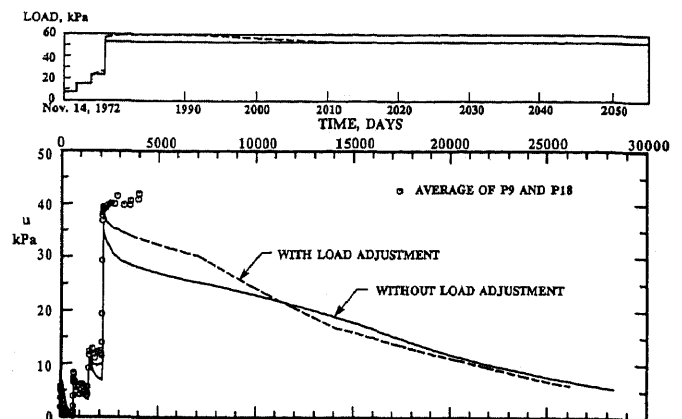


Fig. 16 Observed and Computed Excess Pore Water Pressure at 10 m Depth

Conclusions

Reasonable agreement between the computed and measured settlement and pore water pressure behavior at Ellingsrud confirms ILLICON as being a practical method for settlement analysis of wide foundations and embankments on soft clays. The analysis of Ellingsrud test fill points out the important input data that are required for an accurate prediction of the magnitude and rate of settlement and pore water pressure dissipation.

The Ellingsrud test fill was well instrumented and the incremental loading procedure is extremely valuable. Yet significant uncertainties exist in relation to the nature of soil below 14 m depth and only limited consolidation and permeability tests were performed on good quality samples for rest of the subsoil. It was possible to make reasonable assumptions with respect to the missing subsurface information and obtain good agreement between the predictions and field observations of pore water pressure and settlement. Yet, since there are serious gaps and scatter in the data, one cannot be completely certain that the input parameters in fact represent the true condition and behavior of the soil in the field, and it is not possible to rigorously evaluate the fundamental assumptions on which the ILLICON methodology is based. For example, the computed excess pore water pressures after the final loading at the mid-depth of the stratum show a definite rate of dissipation which is in contrast with the measurements which actually increase. This discrepancy may be related to the fundamental assumptions involved in the formulation of the constitutive equation. Thus, the conclusion is that for test structures in general and test fills in particular to have a major impact on our understanding of soil behavior and analysis procedures, the defining of subsurface conditions should receive as much attention and care as the construction of fill, field instrumentation, and field observations (Mesri and Lo, 1986 and 1989).

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